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2.6 CANAL ROUTING

2.6.1 Basic Principles

Canal or channel flow routing in the South Florida Water Management Model uses a mass balance approach to account for any changes in storage within a canal reach given beginning-of-day canal stage, canal and structure properties, and calculated or specified inflows and outflows. The mass balance is performed every time step (1 day) for each canal reach and involves grid cells through which each canal reach passes. The SFWMM assumes that the width of a canal is constant along its entire length. The model includes the ability to assume either a constant wedge-shaped longitudinal canal profile or a dynamic (daily) wedge-shaped longitudinal canal profile. In both cases, the approximating channels are assumed to be rectangular with a linear slope. In the case of the dynamic slope, the slope in the canal is calculated on a daily basis. The two cases will be discussed below [refer to Section 2.6.2 Profile Slopes in Canals].

The components of the canal water budget are rainfall, evapotranspiration, overland flow (cell-to-canal or canal-to-cell), canal seepage, and structure inflows and outflows. Because some of these components are functions of canal stage, an iterative procedure is used to calculate the end-of-day canal stage.

Rainfall into a canal reach varies by grid cell. The volume of precipitation within a canal segment is equal to the depth of rainfall assigned to a particular grid cell multiplied by the surface area of the canal reach located within the grid cell. Evaporation depth within the canal segment is equal to the product of reference crop evapotranspiration rate and open-water coefficient (KMAX) assigned to the grid cell. A canal segment is that portion of a canal reach that falls entirely within a grid cell while a canal reach is a series of canal segments bounded by the canal's primary inlet and outlet structures.

Canals also interact with freewater or ponded water within the grid cell. In contrast to cell-to-cell overland flow, this interaction provides a means for the model to direct runoff from individual grid cells into canals or to account for overbank flow. Runoff enters the canal as lateral sheetflow, and in situations where excessive canal stages occur, water overtops its banks and becomes part of ponded water. The same approach used to model the resistance to flow for overland flow (refer to Section 2.4) is used to calculate the exchange of canal water with ponded water. Assuming that the canal bisects a grid cell into two 1-mile by 2-mile strips, the slope of the energy grade line, which runs perpendicular to either side of the canal, is assumed to be equal to the ponding depth (not the difference between the stage in the canal and the average stage in the grid cell) divided by one-half of the short side of either strip. This rough approximation, which is equal to one-fourth the length of one side of a grid cell or 0.5 mile, yields satisfactory results. As in the case for cell-to-cell overland flow, the effective roughness coefficient, N, in cell-to-canal (or vice versa) overland flow is expressed as a function of ponding depth at the grid cell where a canal segment is located. N varies considerably with vegetation or land use type. Based on these flow characteristics, different values of parameter A and b can be used as in Equation 2.4.2.8. N may also vary as a function of channel properties such as sediment distribution and riverbank irregularities. This level of detail is not accounted for in the model.

The SFWMM v5.5 has the capability to specify overland flow coefficients which are unique to a specific canal and override those specified in Table 2.6.1.1 which are based on grid cell land use.

Table 2.6.1.1 Values of Parameter A and b Used to Define N for Cell-to-Canal or Canal-to-Cell Overland Flow in the South Florida Water Management Model

	Land Use/Description	A Cell-to- canal	b Cell-to- canal	A Canal-to- cell	Threshold Ponding Depth for SW-Canal Interaction to Start
1	Urban/low density	0.50	0.0	0.25	0.50
2	Agriculture/citrus	0.50	0.0	0.30	0.45
3	Wetland/freshwater marsh	2.00	-0.77	0.30	0.20
4	Wetland/sawgrass plains	2.00	-0.77	1.10	0.25
5	Wetland/wet prairie	2.00	-0.77	1.20	0.25
6	Rangeland/shrubland (scrub and shrub)	2.00	-0.77	1.50	0.25
7	Agriculture/row (or truck) crops	0.50	0.0	0.30	0.45
8	Agriculture/sugar cane	0.50	0.0	0.25	0.09
9	Agriculture/irrigated pasture	0.35	0.0	0.25	0.40
10	Wetland/stormwater treatment area and above-ground reservoir	2.00	-0.77	1.30	0.20
11	Urban/high density	0.25	0.0	0.20	0.45
12	Forest/forested wetlands	2.00	-0.77	0.50	0.10
13	Forest/mangroves	2.00	-0.77	1.00	0.10
14	Forest/melaleuca	2.00	-0.77	0.60	0.25
15	Wetland/cattail	2.00	-0.77	1.00	0.25
16	Forest/forested uplands	1.50	0.0	1.00	0.25
17	Wetland/Ridge & Slough I	2.00	-0.77	1.00	0.25
18	Wetland/marl prairie	2.00	-0.77	0.75	0.25
19	Wetland/mixed cattail / sawgrass	2.00	-0.77	1.00	0.25
20	Water/open water (deep excavated reservoirs)	0.01	0.0	0.01	0.00
21	Wetland/Ridge & Slough II	2.00	-0.77	0.80	0.10
22	Wetland/Ridge & Slough III	2.00	-0.77	1.50	0.25
23	Wetland/Ridge & Slough IV	2.00	-0.77	1.50	0.25
24	Wetland/Ridge & Slough V	2.00	-0.77	1.50	0.25
25	Urban/medium density urban	0.45	0.0	0.25	0.48

Note: The b values for canal-to-cell flow are zero.

Land use types 7, 8 and 9 are the three predominant land use classifications in the EAA. Since overland flow is not simulated in the EAA (refer to Section 3.2), the coefficients corresponding to these land use types are not used in the model.

Canal seepage describes the interaction of canals with the water table (refer to Section 2.5). The operations of structures are site-specific and are discussed, as necessary, throughout this documentation.

2.6.2 Profile Slopes in Canals

The most common assumption in the model for a canal water surface slope is a constant longitudinal profile such that a constant offset or head drop, HDC, occurs along the entire length of each canal (Figure 2.6.2.1). In South Florida, both the beginning and end stage of the canals are often monitored and a representative slope can be determined from observed conditions. The constant offset can be considered as a pre-defined slope in the hydraulic grade line that represents the average or long-term difference between the stage in the canal at its upstream end and at its downstream end. For a given constant slope canal, HDC is specified as two values, one for the dry season and one for the wet season. This slope remains unchanged from one time-step to the next (except for the change from season to season) and is independent of the discharge in the channel. When a canal reach spans more than one model grid cell, the total head drop over the canal reach is assumed to be evenly distributed across the cells (i.e. change in head from one segment to another is uniform).

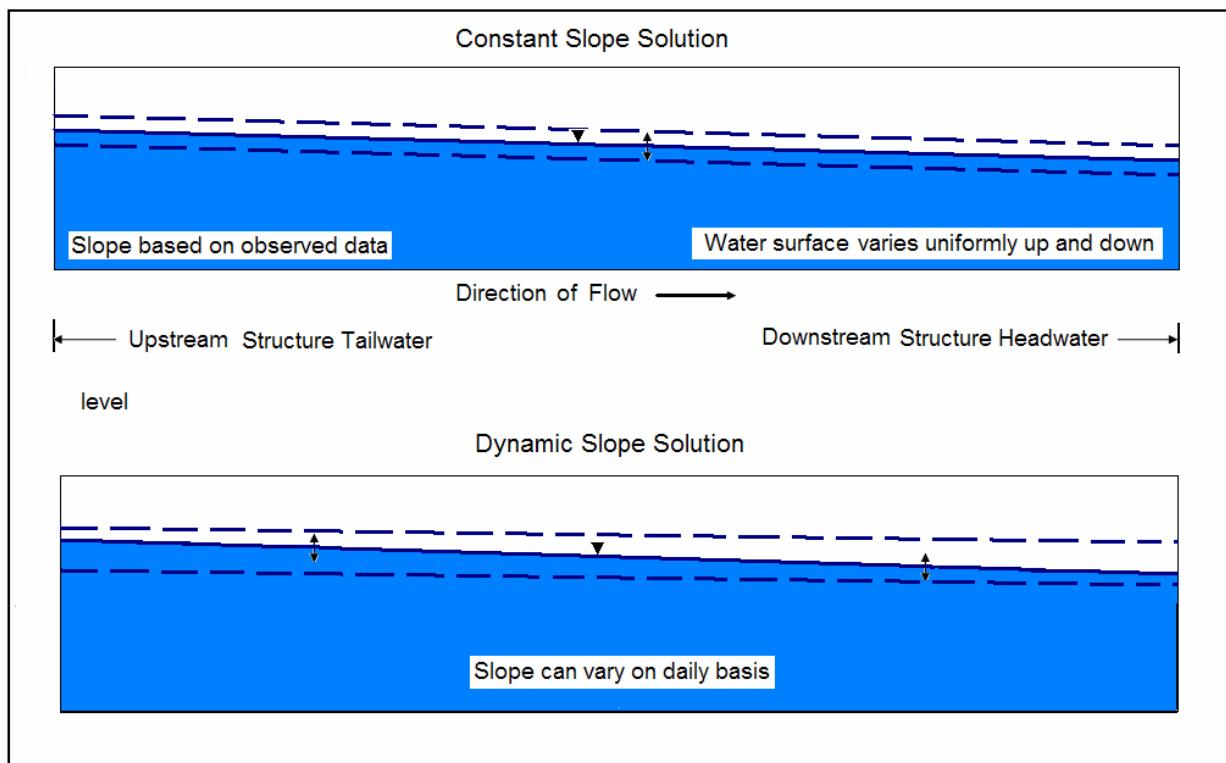


Figure 2.6.2.1 Canal Profiles Showing Head Drop

In some of the LEC canals where sufficient calibration data exists, a dynamic longitudinal profile is calculated such that a time varying offset or head drop occurs over the length of the canal reach (Figure 2.6.2.1). Alternately stated, this means the total HDC varies from one time-step to the next. For any given time step, head drop from segment to segment within the reach still remains uniform. The targeted canals are mostly in the LEC, urban, more-developed areas that are less affected by ponded cell water than canals in other parts of the system.

In calculating dynamic canal slopes, the effects of both inflow and outflow (and where it occurs along the canal) are considered as part of the calculation of head drop. In order to apply Manning's equation when calculating slope, one representative effective flow value must be derived for the entire reach. To calculate this flow term, the SFWMM uses a simplified weighting scheme in which all of the in-line and lateral inflows and outflows are considered. This methodology is graphically illustrated in Figure 2.6.2.2. First, the canal is divided in half and then the accounting of flows is completed for both segments by applying Equation (2.6.2.1). The most downstream segment is considered to be the first segment. The sign conventions of inflows and outflows play an important role in determining the canal slope. The upstream in-line and lateral flows coming in and the downstream in-line and lateral flow going out have a positive sign meaning that they would contribute to increasing the slope.

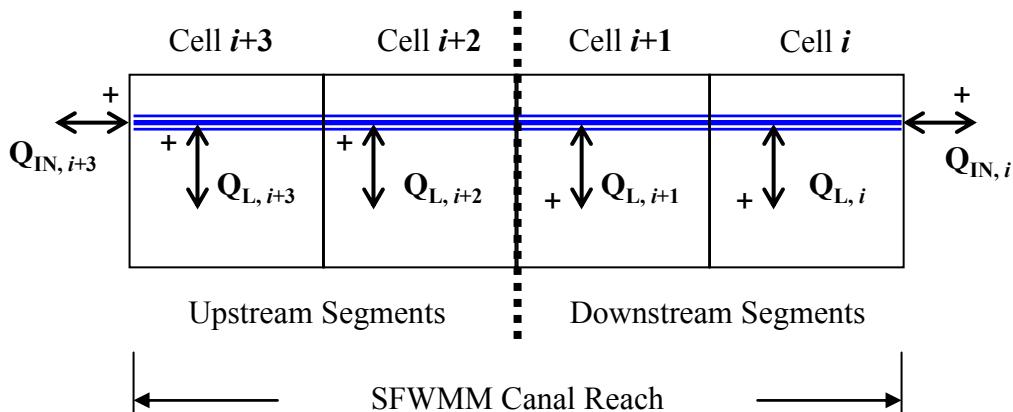


Figure 2.6.2.2 Conceptual Diagram for Calculating Flow in Determination of Slope for Dynamically Dimensioned Canal Reaches

$$Q_{EFF} = (Q_{IN,n} + Q_{IN,1})/2 + (\sum_1^{n/2} Q_L)/4 + (\sum_{n/2+1}^n Q_L)/4 \quad (2.6.2.1)$$

where:

Q_{EFF} = Effective flow to be used in determination of canal slope.

n = Number of segments forming the canal reach.

Q_{IN} = Net in-line structural flow term calculated at most upstream and downstream segments.

Q_L = Net lateral flow term calculated at all canal segments. This term is made up of overland and groundwater (seepage) flows as well as structure flows.

Because the head-drop calculation is part of the iteration of the canal solution, a limit is specified on both the maximum and minimum head-drop (based on historical values) so the solution for

flow and stage does not iterate beyond normal operating limits. The dynamic canal slope method has limited applicability because it does not work well in areas where: (1) it is difficult to quantify the inflows and outflows; and (2) where ponding occurs in the canal most of the time (e.g. in WCAs).

In addition to the dynamic slopes, another major influence on how canals respond to different flow situations is linked to actual observed structure operations. At each structure a single set of on/off triggers is normally relied upon for operations, but lower gate settings may be used in certain high flow events that are triggered by local rainfall accumulation. Input to the model allows canals to respond to different rainfall thresholds. If the 14-day average rainfall exceeds a threshold input, then the gate would be lowered by a slight deviation from the normal operations. If the 14-day average rainfall exceeds a second threshold input, then the gate would be lowered by an additional amount. These short term operational deviations help modeled canal profiles to respond in a manner similar to that of observed field data.

2.6.3 Canal Water Budget

The beginning-of-day stage CHDEP⁰ at the most downstream node of a canal reach is set equal to the stage at the end of the previous time step. The canal stage at the most upstream node of the same canal reach is set equal to the stage at the most downstream node plus offset HDC. Canals defined at intermediate nodes are assumed to have stages proportional to their relative distances from the extreme nodes of the reach.

The initial estimate of the end-of-day or equilibrium stage (CHDEP⁰) at the downstream node is assumed to be equal to beginning-of-day stage. Initial change in storage CHSTOR⁰ is, therefore, zero. Rainfall and evapotranspiration are calculated for each canal segment using methods described above. Discharge at the downstream structure (typically a weir or pump) is calculated as a function of its headwater (= CHDEP⁰). Discharges elsewhere within the reach (other outlet structures, canal seepage and overland flow) are either prescribed (e.g., historical) or calculated as a function of CHDEP⁰ adjusted for their location relative to the most downstream node of the reach and the slope of the assumed constant or dynamic hydraulic grade line HDC. In dynamic slope canals, the previous end-of-day HDC is assumed as the initial estimate of slope prior to iteration.

The net inflow or accumulation ACVOL⁰ is calculated using

$$ACVOL^0 = Q_{in} - Q_{out} \quad (2.6.3.1)$$

where:

$$\begin{aligned} Q_{in} &= RF + OVLNF_{in} + SEEP_{in} + QSTR_{in}, \text{ and} \\ Q_{out} &= ET + OVLNF_{out} + SEEP_{out} + QSTR_{out}. \end{aligned}$$

It should be noted that OVLNF_{in}, OVLNF_{out}, SEEP_{in}, and SEEP_{out} are functions of the assumed end-of-day stage. Therefore, they are implicit functions of the unknown stage. QSTR_{in} and QSTR_{out} may or may not be implicit functions. Solving for the change in storage based on beginning- and assumed end-of-day stages:

$$CHSTOR^0 = [(CHDEP^0 + HDC^0/2) - (CHDEP0 + HDC0/2)] (CAREA) \quad (2.6.3.2)$$

where CAREA is the surface area of the canal reach equal to the product of the width and the length of the canal reach. Note that in constant slope canals, where HDC^0 equals $HDC0$, slope cancels out of the change in storage calculation.

By definition, canal water budget indicates that the change in storage $CHSTOR^i$ must be equal to the net inflow or accumulation $ACVOL^i$, where i denotes the i^{th} iteration within the same time step. The difference is the estimation error given by:

$$ERROR^0 = CHSTOR^0 - ACVOL^0 \quad (2.6.3.3)$$

Eliminating this error is the objective in establishing a canal water budget. The objective is met by iteratively assuming the end-of-day stage. A positive error implies that the assumed end-of-day stage is overestimated. In order for the canal to experience a change in storage $CHSTOR^0$ due to $CHDEP^0$ more inflow (or less outflow) should have resulted from the same $CHDEP^0$. Thus, if $CHSTOR^0 > ACVOL^0$, the new estimate at the downstream stage is made lower than the previous estimate, $CHDEP^1 < CHDEP^0$. Conversely, if $CHSTOR^0 < ACVOL^0$ the new estimate is raised, $CHDEP^1 > CHDEP^0$. $CHDEP^0$ is incremented (decremented) to $CHDEP^1$ based on the magnitude of the error, an initial increment value (INC^0) of 1.0 ft is used. Therefore,

$$CHSTOR^1 = CHSTOR^0 + INC^0 \quad (2.6.3.4)$$

The calculations enter an iteration loop where Equations 2.6.3.4 and 2.6.3.1 through 2.6.3.3 are solved and a stopping criterion is tested. If the value of $ERROR$ changes sign, the magnitude of the increment in stage is halved to prevent oscillation between successive stage estimates. (The number of iterations is the number of times the assumed equilibrium stage is updated.) The iteration loop is terminated when either of the following stopping criteria is met: (a) the absolute value of $ERROR/CAREA$ becomes less than the convergence value (0.01 ft); or (b) the maximum allowable number of iterations (110) has been reached. In general, constant slope canals are able to converge more quickly than dynamic slope canals due to the above mentioned simplification of Equation 2.6.3.2. In the event that a dynamic slope canal is unable to converge within forty iterations, downstream segment stage is fixed (this term habitually becomes stable by this number of iterations) and slope is determined using a standard-step iterative approach based on the last two HDC values as calculated by the Manning's approximation. This method determines the final HDC subject to the above mentioned global constraints.

The above calculations are performed for all canal reaches in the model domain except for those in the EAA. Conveyance considerations in the major EAA canals are discussed in Section 3.2.